

Micro-structure and Engineering Behaviour of Weakly Bonded Soil

(Mikro-struktur dan Sifat Kejuruteraan Tanah Berikatan Lemah)

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ABSTRACT

This study was carried out to investigate the undrained behaviour of weakly bonded soil. Artificial specimens were used and were prepared from a mixture of sand and kaolin. The mixture was fired at 500°C for 5 h to create uniform strength of artificial weakly bonding between sand particles. Based on microscopic observation of the specimens revealed that the fired kaolin formed bridges between the sand particles and most of the sand particles were entirely coated by kaolin. It was observed that very few sand grains made a direct grain to grain contact. The occurrences of inter-particle voids either isolated or connected to form larger voids. Fissures found in between the bonding and the grains were due to high temperature firing. A series of consolidated undrained tests (CU) were performed using conventional triaxial compression test. The stress ratio curves for the bonded specimens showed a significant peak at early strain and the values of maximum stress ratios dropped with increasing applied confining stresses. For the destructured (or unbonded) specimens, no apparent peaks were observed and the stress values were slightly lower compared to that of bonded specimens. The bounding surface for the destructured specimens is represented by a straight line equivalent to stress ratio of 1.24. This line is equivalent to strength parameters of internal friction angle, ϕ' and cohesion intercept, c' of 31° and 21 kPa, respectively. Meanwhile, the bounding surface for the bonded specimens is indicated by some curvature at low stress ($p' < 150$ kPa) and high stresses ($p' > 1100$ kPa). The study clearly suggested the role of bonding on the engineering behaviour of residual soils in undrained conditions.

Keywords: Bonded soil; residual soil; undrained tests; void ratio

ABSTRAK

Kajian telah dilakukan bagi menyiasat sifat tidak bersalir bagi tanah berikatan lemah. Spesimen buatan telah digunakan dan disediakan daripada campuran pasir dan kaolin. Campuran tersebut dibakar pada suhu 500°C selama 5 jam untuk menghasilkan ikatan lemah buatan dengan berkekuatan sekata antara butiran pasir. Pemerhatian mikroskop terhadap spesimen mendedahkan pembentukan jejambat kaolin di antara butiran pasir dan kebanyakan partikel pasir disaluti oleh kaolin. Sedikit persentuhan terus antara butiran ke butiran diperhatikan. Kehadiran reruang antara butiran sama ada terasing atau berhubungan membentuk reruang yang lebih besar. Rekahan-rekahan yang ditemui di antara pengikatan dan butiran adalah disebabkan oleh kesan pembakaran pada suhu yang tinggi. Siri ujian terkukuh tidak bersalir (CU) telah dilakukan menggunakan ujian konvensional mampatan tiga paksi. Lengkok-lengkok nisbah tegasan bagi spesimen berikatan menunjukkan puncak yang ketara pada awal terikan dan nilai-nilai maksima nisbah tegasan menyusut dengan peningkatan tegasan mengurung yang digunakan. Bagi spesimen tak berstruktur (tidak berikatan), tiada puncak yang jelas diperhatikan dan nilai-nilai tegasan adalah lebih rendah berbanding spesimen berikatan. Permukaan sempadan bagi spesimen tidak berikatan diwakili oleh garis lurus setara dengan nisbah tegasan 1.24. Garisan ini setara dengan parameter kekuatan bagi sudut geseran dalam, ϕ' dan pintasan jelek, c' masing-masing sebanyak 31° dan 21 kPa. Manakala permukaan sempadan bagi spesimen berikatan pula ditunjukkan oleh sedikit lengkungan pada tegasan rendah ($p' < 150$ kPa) dan tegasan tinggi ($p' > 1100$ kPa). Kajian ini jelas mencadangkan peranan pengikatan di dalam lakuan kejuruteraan tanah baki dalam keadaan tidak bersalir.

Kata kunci: Nisbah liang; tanah baki; tanah berikatan; ujian tidak bersalir

INTRODUCTION

Soils are developed from various geological origins and associated processes such as weathering, erosion, transportation and sedimentation. The *in-situ* weathering of parent rocks is due to chemical and/or physical processes finally generating residual soils. Since residual soils have been formed from *in-situ* weathering, their engineering characteristics are much influenced by stress history,

grain strength, bonding, relict discontinuities, anisotropy and void ratio (Blight 1997). The presence of bonding or cementation has been recognised as part of the soil structure. This bonding is commonly found in a wide range of earth materials which are considered as engineering soils in geotechnical engineering, such as soft and stiff clays, granular soils and residual soils (Leroueil & Vaughan 1990). Burland (1990) introduced the term “structure”

for clays which were studied at a microscopic scale. This structure refers to the combination of fabric and bonding. Meanwhile, for granular materials such as sands, this structure mainly refers to the occurrence of cement between the particles; however, little emphasis has been given to the fabric. Taylor (1948) mentioned that the strength of granular soils is partly controlled by the friction among particles and particle interlocking. Schofield and Wroth (1968) extended the theory and proposed a following equation $q'/p' = M - (\delta\varepsilon_v/\delta\varepsilon_s)$, where q'/p' is the stress ratio, M is the gradient of critical state line when the rate of volume change is constant, $\delta\varepsilon_v$ is a small increment in volumetric strain and $\delta\varepsilon_s$ is a small increment in shear strain ($-\delta\varepsilon_s$ indicates an expansion). The relationship between stress ratio, q'/p' and rate of rate of dilation, $\delta\varepsilon_v/\delta\varepsilon_s$ was explained by Atkinson and Barnsby (1978). The q'/p' ratio achieves a maximum value at which the sample expands at the maximum rate of dilation during shearing. Once the stress ratio, $q'/p' = M$, the change in shape of sample immediately occurs at a constant volume (where $\delta\varepsilon_v/\delta\varepsilon_s = 0$). Beyond this point ($q'/p' \geq M$), the sample then continues to dilate and achieves the maximum q'/p' ratio at maximum rate of dilation before it gains its critical state. Since much work has been concentrated on clean sands, more recently the direction has been changed to focus on sand with a proportion t of fine contents. According to Yamamuro and Lade (1998), silty sands (with 7% non-plastic Nevada sands) are the most common type of soil associated with both static and earthquake-induced liquefaction. The maximum void ratio was 0.87 and the minimum void ratio was 0.55. A "reverse" sand behaviour was observed for the tests performed at low confining stress, because sands normally exhibit the opposite trend of increasing instability with increasing confining stress. Lade and Yamamuro (1997) and Zlatovic and Ishihara (1997) found that the presence of significant silt content in sand deposits makes them more prone to liquefaction than clean sands.

Much research has been carried out on various kinds of soils in order to understand their engineering behaviour (Vaughan 1988; Vaughan & Kwan 1984). Due to the difficulties in sampling, transportation and storage of natural soils, artificial soils have also been used widely to simulate the structure of natural soils. Natural soils are heterogeneous and pose variability in void ratio and bond strength (Vaughan et al. 1988). The properties of artificial soils can be controlled in the laboratory; therefore permitting their engineering behaviour can be more easily studied. Due to the difficulties in handling of natural soils and the advantages offered by artificial specimens, much research in fundamental framework of soil mechanics have been developed from reconstituted or artificial specimens (Bressani 1990; Coop & Atkinson 1993; Haeri et al. 2006; Huang & Airey 1991; Maccarini 1987). Maccarini (1987) developed a technique for preparing artificial soil in order to simulate the characteristics of residual soil.

Maccarini used 13% of kaolin, 30% of weak sand-sized particles obtained from crushed fired kaolin and 5% of quartz sands. Bressani (1990) used same material

adopted from Maccarini (1987) but prepared with different proportions of material and bond strength. Meanwhile Huang and Airey (1991) and Coop and Atkinson (1993), studied the artificial specimens of cemented carbonated sand in order to investigate the effects of cementation and density on the stress yield. The results were also compared between cemented and uncemented carbonated sands. The effects of different types of cements were examined by Haeri et al. (2005). Specimens were prepared by mixing a base soil with Portland, gypsum and lime to represent different types of cementation agents.

The properties of the artificial soils should be reproducible and should not change with time. In this study, a series of undrained triaxial compression tests were conducted on artificially prepared specimens for bonded and destructured soils. The objectives of this work were to investigate the micro fabric of the prepared artificial specimens and the affect of weakly bonded structure on the engineering behaviour in undrained conditions. The results from this study would be used as references for the study of a similar material in saturated drained tests and unsaturated constant water tests in developing the framework model for unsaturated soils.

METHOD

PREPARATION OF ARTIFICIAL SPECIMENS

In this study the engineering behaviour of soil was studied using artificial specimens. The reason of using artificial specimens was to simulate the presence of weak bond structure of residual soils. Although the bond strength in residual soils is particularly low, its contribution to shear strength is important.

Two states of soil specimens were prepared namely bonded and destructured soils. For the specimens used for the undrained tests, values of initial void ratio and applied confining pressures are summarised in Table 1. These bonded and destructured specimens were prepared from the mixture of sand and kaolin. The sand adopted was Leighton Buzzard sand which is classified as a uniformly medium sand (Figure 1).

A similar technique was initially adopted by Maccarini (1987) and Bressani (1990) then followed by Malandraki (1994). In this study preparation of artificial specimen was carried out according to the technique adopted by Malandraki (1994). Cylindrical specimens with dimensions of 38 mm diameter and 76 mm height were used in this study. The ratio of sand to kaolin in the mixture (by dry weight) was 87:13. A small fraction of kaolin (13% of total mixture) were used in this mixture was to establish weak bonding between sand particles resulting from the fired kaolin (Malandraki 1994). Clean dry sand and kaolin were initially mixed before distilled water was added. The mixture was thoroughly stirred and transferred into a steel mould lined with filter paper. In order to achieve uniform distribution of material across the specimen, about 10 g

of wet mixture was allocated to each layer. Each layer was gently tamped and levelled, using a plastic piston up to 15 layers. The wet mixture was left to dry at room temperature. All specimens have a similar void ratio of approximately 0.6 and uniform bond strength. The

TABLE 1. The undrained tests on bonded and destructured specimens

Tests	Initial void ratio, e_o	Applied confining pressure, p_o' , kPa
(a) Bonded Specimens		
cub25	0.6100	25
cub30	0.6015	30
cub50	0.6077	50
cub100	0.6135	100
cub200	0.6083	200
cub300	0.6044	300
cub400	0.6012	400
cub500	0.5998	500
cub600	0.6035	600
cub700	0.6015	700
cub800	0.6081	800
cub900	0.6085	900
cub1000	0.6035	1000
(b) Destructured specimens		
cud25	0.6048	25
cud60	0.6103	60
cud100	0.6018	100
cud200	0.6048	200
cud300	0.6103	300
cud400	0.6097	400
cud500	0.6022	500
cud600	0.6152	600
cud700	0.6092	700
cud800	0.6047	800
cud1000	0.6113	1000

uniform bond strength can be achieved from the firing of specimen at a constant temperature and duration (Grim 1953). The dried specimen was then fired in a furnace at 500°C for 5 h (Figure 2). Representative thin sections were prepared for the microscopic observation. In preparing the thin sections, the specimens were initially immersed in acetone before being impregnated using Araldite resin under vacuum for about 48 h at 50°C. Dye was also added to the solution in order to colour the pores within the samples. A Nikon YS100 optical microscope was used and has the capability of magnifying up to 40×. The thin sections were photographed using the microscope equipped with a camera.

The destructured soils were prepared in the same manner as bonded soils. The bonded soils were broken down manually by hand to destroy the bonds and were reshaped using the same steel mould in order to achieve a similar composition and grading. The void ratio of destructured soils was prepared at same void ratio as the bonded soils for comparison of their engineering behaviour with the bonded soils. Therefore, the effect of bond structure of the studied soils could be investigated and assessed.

EXPERIMENTAL PROCEDURES

A series of undrained triaxial compression tests were carried out on 13 specimens of artificially weakly bonded soils and 11 destructured soils. The specimens were initially saturated by application of back pressure in order to achieve a degree of saturation (B value) of at least 0.95. Since the specimens were considerably porous, the targeted degree of saturation would be achieved within 24 h. Then the specimens were consolidated with different effective stresses ($p-u_w$) of 25 kPa up to 1000 kPa. Each specimen was allowed to consolidate under the applied confining pressure for an hour using the computer control system

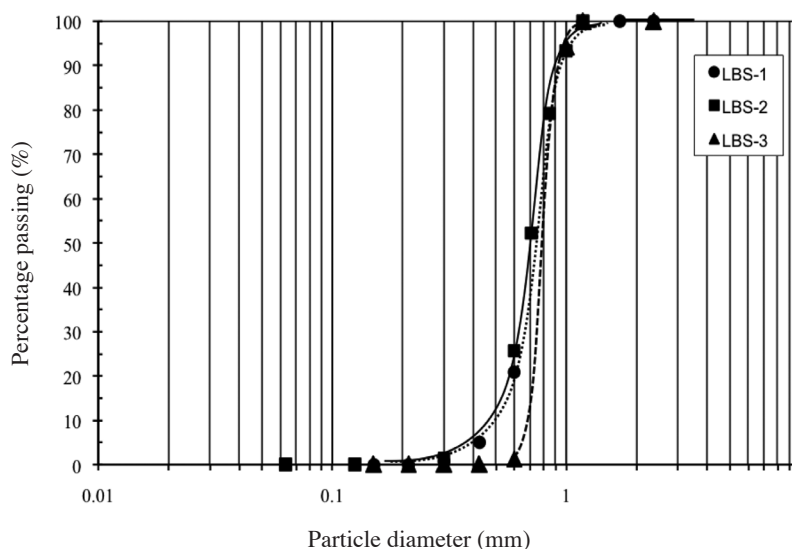


FIGURE 1. Particle size distribution curves for Leighton Buzzard sands (LBS) used in this study



FIGURE 2. The artificial specimens after fired at 500°C for 5 hours in furnace with void ratio of 0.6

(TRIAX ver. 5.0) prior to shearing. The shearing of the soil specimens was carried out at constant rate of strain at 0.025%/min under undrained conditions and shearing was continued up to 35% of strain in order to establish the behaviour of specimens towards the critical state. The destructured soil specimens were also examined under similar undrained conditions in order to compare the results with those for the bonded soil specimens. A detail of experimental procedures can be referred to Ali Rahman (2008). The results of the shearing of deviator stress, q , excess pore water pressure, Δu and stress ratio, q/p' were plotted against the axial strain, ϵ_a (%). The stress paths and bounding surfaces of the bonded and destructured specimens were presented in q - p' spaces. The identification of the tests was based on the type of test, condition of sample and applied confining stress prior to shearing. The first two letters “cu” refer to the consolidated undrained condition during shearing while the third letter “b” or “d” indicates whether a bonded type of sample or a destructured sample was being used for the test. Finally the numbers refer to the value of confining stress used during isotropic consolidation of the sample.

RESULTS AND DISCUSSIONS

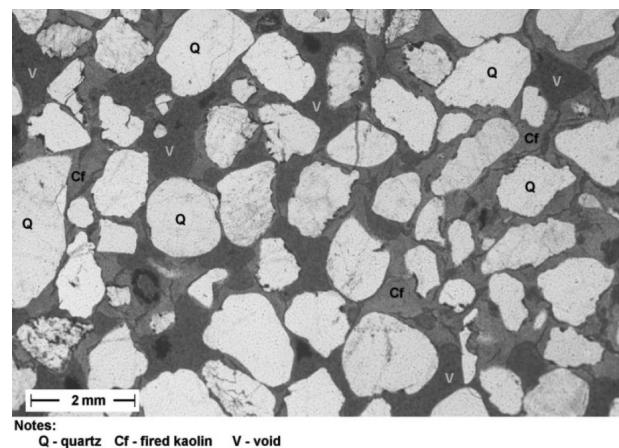
MICROSCOPIC OBSERVATION

Observation of the soil fabric was performed with the aim to study the physical arrangement of grains and macrostructure of the prepared specimens. Figures 3a and 3b were taken from two different samples with void ratio of 0.6 under $\times 2.5$ magnification. The quartz sand particles were identified as white grains meanwhile the fired kaolin could be traced from its light grey appearance and usually occurred close to quartz particles. The majority of the quartz grains are classified as sub-angular to sub-rounded grain according to the IAEG classification (Anon. 1979). The black areas are identified as voids. It was clearly seen from the photographs that the fired kaolin established

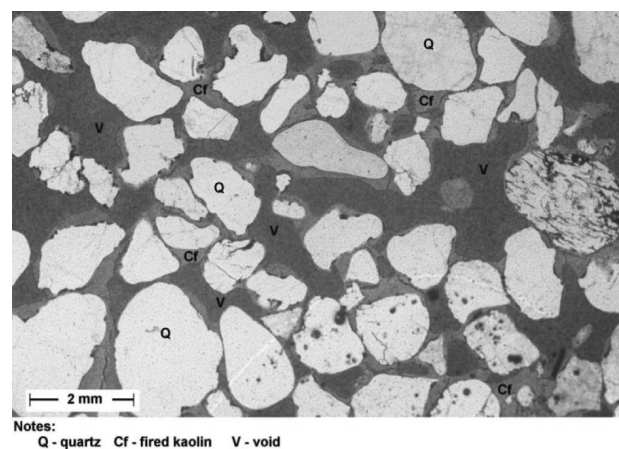
bridges of bonding whereas in some part of the section, it coated the whole sand particles (Figure 3a; right side). It also revealed that very few sand grains had direct grain to grain contacts. The occurrences of voids can be isolated (when surrounded by fired kaolin) or connected to each other to form larger voids (Figure 3b). A number of fissures were detected in between the bonding and the grains. Since the samples were fired at higher temperature followed by cooling, this could be due to the fired kaolin contracting and causing the development of fissures. A similar feature was also reported by Bressani (1990) who used similar materials for artificial bonded samples.

STRESS-STRAIN AND PORE WATER PRESSURE BEHAVIOUR

The deviator stress, q and excess pore water pressure, Δu against axial strain curves for bonded and destructured soils are shown in Figure 2 and 3. Since the tests were carried out in the undrained state, the shearing of samples is associated with changing in pore water pressure, u . Positive and negative pore water pressures correspond to the compression or dilation in terms of volumetric change in drained tests.



(a)



(b)

FIGURE 3. Microscopic photographs of artificially weakly bonded sample with void ratio 0.6 under $\times 2.5$ magnification

The stress-strain curves for bonded and destructured specimens showed an initial linear increase of q at early strain, ϵ_a of about 2%. (Figures 4a & 5a). It is clearly seen that the maximum values of q increases with the increase in applied confining stress, p' . All bonded soils showed a brittle failure due to development of shear plane where the stress ratio begins to decrease with axial strain. Meanwhile the modes of failure for the destructured soils were represented by development of shear plane and bulging. Test for cud25, cud60, cud200 and cud600 (destructured specimen) showed bulging failure mode as the q gradually increased up to the end of shearing (Figure 4).

In terms of pore water pressure, both states of soils (bonded and destructured) showed a development of positive pore water pressure at early stage (less than 5% of axial strain) of shearing and reached the maximum value before decreasing up to a particular axial strain (Figures 4b & 5b). For the bonded soils which were sheared at lower p' (≤ 500 kPa), a negative excess pore water pressure, Δu was observed due to dilation behaviour of the specimens.

However for specimens sheared at higher p' (≥ 500 kPa), specimens have a tendency to compress up to the end of shearing as indicated by gradual increase in Δu . It is clearly exhibited that the increase in the amount of initial positive Δu also corresponded to an increase in applied effective confining stress. It was also found that the Δu was still changing up to the end of tests suggesting that the soils did not achieve their true critical state or this was probably caused by the strain localisation.

STRESS RATIO, q/p'

The stress ratio, q/p' against strain curves for the bonded and destructured soils are shown in Figure 4. Stress ratios for both types of soils were studied at three different sets of confining stresses; low ($p'_o = 25, 30, 50$ and 60 kPa), intermediate ($p'_o = 300, 400$ and 500 kPa) and high ($p'_o = 900$ and 1000 kPa). The bonded tests indicated by rapid linear increase in q/p' ratio and this only true for destructured samples at low confining stresses. The destructured sample sheared at medium and high confining stresses, the increments in q/p' ratios were slightly lower than the bonded samples.

It is apparent that the bonded samples which were sheared at low confining pressures sustain a higher q/p' ratio than that of destructured samples at a similar range of stress levels (Figure 6a). The maximum q/p' ratio for the bonded samples was also achieved at a lower axial strain than that of the destructured samples. The maximum q/p' ratio for cub25, cub30 and cub50 were 2.00, 1.97 and 1.74, respectively. Meanwhile for the destructured soils, the maximum q/p' ratio were 1.50 and 1.68 for cud60 and cud25, respectively. The maximum of q/p' ratio for both bonded and destructured soils were confined at early axial strain of less than 2%. The bonded samples indicated a significant peak of q/p' ratio, while no apparent peak in the stress ratio was observed from the destructured samples. The higher the applied confining stress, the bonded soils

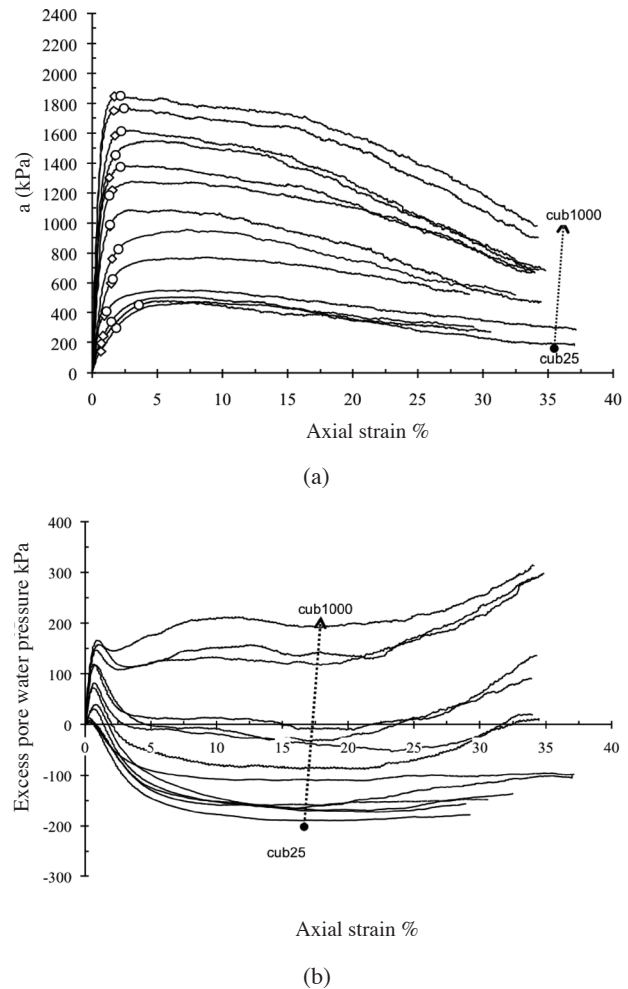
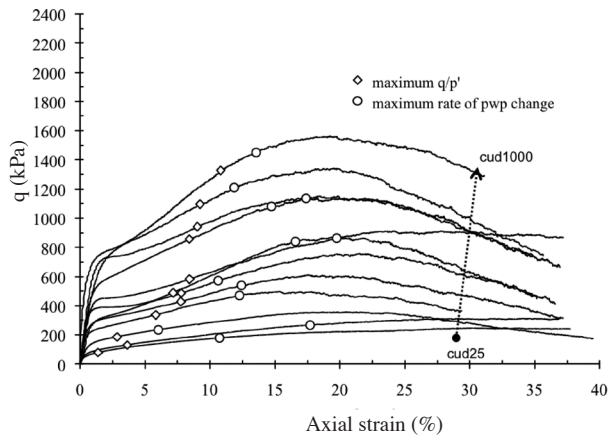


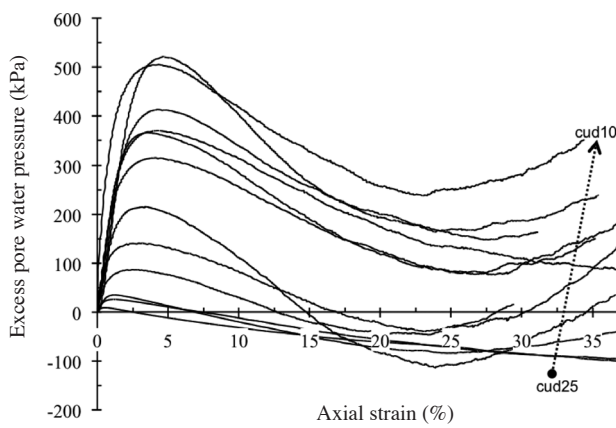
FIGURE 4. Stress-strain for undrained bonded soils. (a) deviator stress vs. axial strain; (b) pore water pressure vs. axial strain

experienced a decrease in the peak of q/p' ratio. At the end of shearing, the bonded samples, cub25 and cub30 showed almost similar stress ratio while cub50 settles at a slightly lower value. Meanwhile the stress ratio values for cud25 and cud60 were indicated by gradual decrease up to the end of shearing but remain higher than the critical state values of the bonded soils.

Stress ratios against axial strain plots for bonded and destructured samples sheared under undrained conditions at medium range of confining stresses ($p' = 300, 400$ and 500 kPa) are shown in Figure 4b. The maximum q/p' ratios for cub300, cub400 and cub500 were 1.41, 1.43 and 1.40, respectively. Meanwhile for the destructured soils, the maximum q/p' ratios were 1.28, 1.24 and 1.28 for cud300, cud400 and cud500, respectively. The maximum q/p' ratios of bonded samples were achieved at a lower axial strain while the destructured soils occur at a higher axial strain ($>5\%$). Bonded samples sheared at this stress levels still show a peak suggesting that the bond still contributes to a higher stress ratio value. However, the overall contribution to the strength becomes lesser as a result of proportion breakdown of bond structure (Vaughan 1988).



(a)



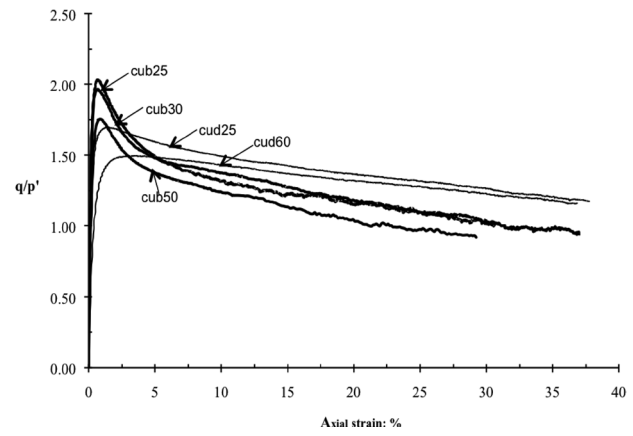
(b)

FIGURE 5. Stress-strain for undrained destructured soils (a) deviator stress vs. axial strain, (b) pore water pressure vs. axial strain

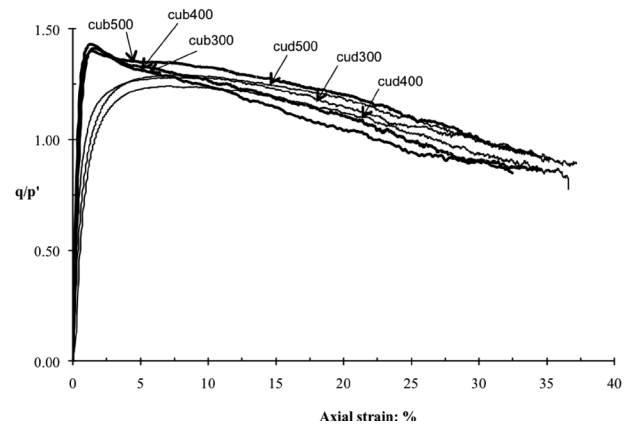
Stress ratios for tests on bonded and destructured samples sheared at high confining stresses ($p' = 900$ kPa and 1000 kPa) are shown in Figure 6c. The maximum q/p' ratio for cub900 and cub1000 were 1.27 and 1.25, respectively and 1.32 and 1.23 for the destructured soils at similar stress levels. At this range of p' , the maximum q/p' ratio of the bonded samples is very close to that of the destructured samples. A similar picture can be seen where the maximum q/p' ratio for the bonded samples is achieved at a lower axial strain than the destructured samples. Toward the end of shearing, the stress ratios for the bonded and destructured samples almost coincide (Figure 6c). The closeness position of the maximum q/p' ratio of two types of soils indicates that the behaviour of bonded soils has become similar to that of the destructured soils at these stress levels. The bond structure of the bonded samples has almost been destroyed at these stress levels and shows a lower stress ratio similar to that of the destructured samples.

STRESS PATHS

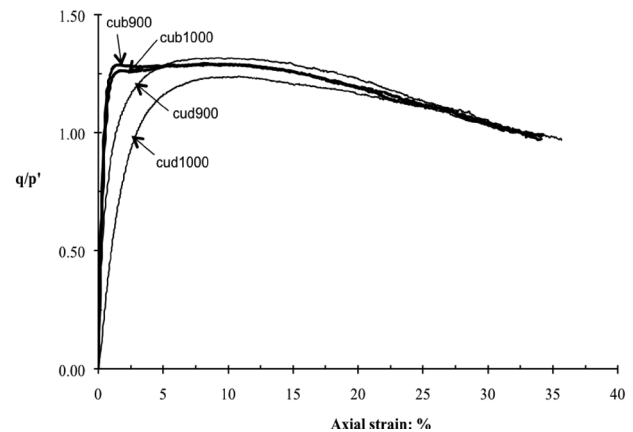
The effective stress paths for both types of soils are presented in Figures 7a and 7b (where $q = \sigma_1 - \sigma_3$ and $p' =$



(a)



(b)



(c)

FIGURE 6. Stress ratios for undrained bonded (cub) and destructured (cud) soils. (a) at low stress, p' (b) intermediate p' and (c) higher p'

$(\sigma_1 + \sigma_2 + \sigma_3)/3$). The positions of maximum q/p' and maximum rate of pore water change were also marked on each curve. Each individual path especially for bonded soils defines a fairly straight line. At lower stress ($p' \leq 300$ kPa) the path linearly increased to reach the maximum q/p' ratio and then changed slightly in direction as q continued to increase. The points of maximum q/p' and maximum rate

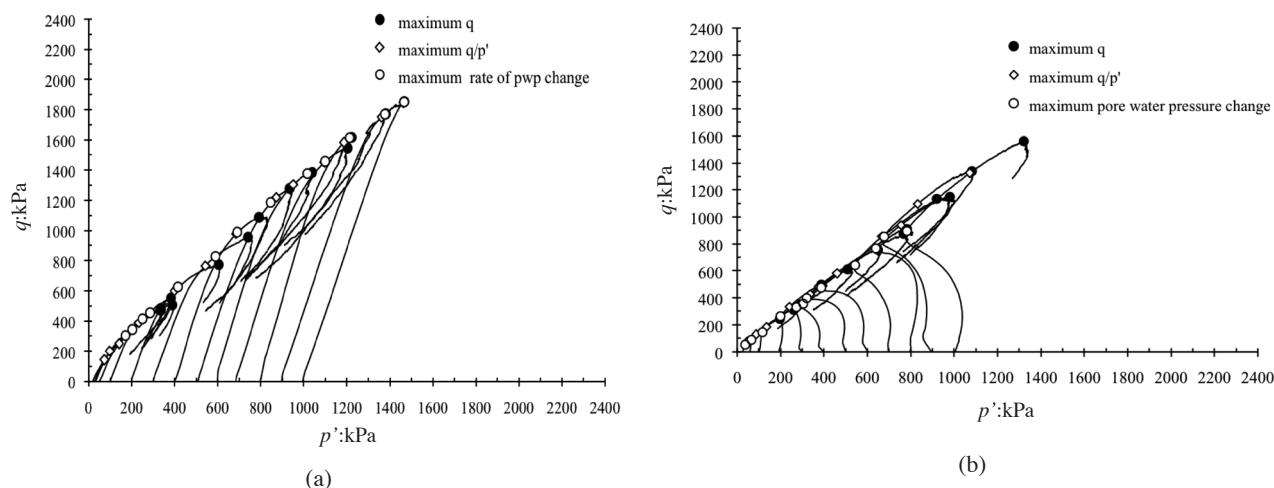


FIGURE 7. Effective stress paths for undrained tests for (a) bonded samples (b) destructured samples

of pore water change are superimposed or closely situated as seen in Figures 7a and 7b. At this state soil achieved its maximum strength as mentioned by Schofield and Wroth (1968). The path reversed its direction after achieving maximum q with decreasing of q and p' . This is most probably associated with the development of shear planes. At higher stresses ($p' \geq 300$ kPa), the maximum q/p' ratio points located away from maximum rate of pore water pressure change. For these tests, the maximum q/p' ratio is achieved at lower deviator stresses than the point at which the maximum rate of pore water pressure change occurs. As the applied stresses increased, the bond structures of the bonded soils gradually destroyed, causing the soil samples sustain lower maximum q/p' ratio than the maximum rate of pore water pressure change.

Meanwhile, stress paths for destructured soils at lower stresses (< 200 kPa) moved to the right corresponding to an increasing negative pore water pressure. At a higher stress level, stress paths tend to move to the left more significantly with increasing confining pressure. After that the paths continue to climb gradually along a fairly constant stress ratio up to failure. The q/p' stress ratio values started to drop significantly whenever shear surface developed in the samples. Some of the samples continue to higher stress along the linear line associated with bulging type of failure with no initiation of shear surface.

BOUNDING SURFACES

The bounding surface was prepared from the maximum q/p' ratio for bonded and destructured soils (Figure 8). It is clearly observed that the bounding surface for bonded soils showed some curvature at high and at lower stress levels. A similar picture was also observed by Malandraki (1994) suggesting that the decrease in the slope of the bounding surface indicates the loss of bond strength of the samples at failure. Meanwhile the bounding surface for destructured soils is defined by almost a linear surface that exist at lower stress compared to the bonded soils. This line

represents the bounding surface for the destructured soils of q/p' value equivalent to 1.24. The equivalent values of angle of friction, ϕ' and cohesion intercept, c' are 30.9° and 21 kPa, respectively.

The bounding surface for the bonded samples turns toward the destructured samples starting at a high stress of about $p' = 800$ kPa and then the two lines coincide at $p' = 1500$ kPa. For a particular range of stresses, it is found that the bonded samples sustain a higher stress ratio compared to the destructured samples. This is clearly as a result of the influence of bonding. At low stress level of about $p' < 150$ kPa, the maximum q/p' ratio values of bonded samples are close to the destructured samples. The destructured samples sheared at this low stress level and achieved a high negative pore water pressure (Figure 7b). However at higher stress level ($p' > 150$ kPa) the bonded samples experienced a higher negative pore water pressure than the destructured samples and shifted up the bounding surface of the bonded samples. Further up to about $p' = 500$ kPa, the bounding surface appears linear for the two samples. For $p' > 500$ kPa the bounding surface for bonded samples again begins to curve up and become linear up to $p' = 850$ kPa. At higher stress ($p' > 850$ kPa) the bounding surface curves down until the lines become coincident at $p' = 1500$ kPa. The bonded samples clearly sustain a higher limiting stress ratio than the destructured samples at intermediate stress levels. However by increasing the stress levels, the strength of the bonds at failure start to degrade and for $p' > 1100$ kPa the bounding surface begin to turn toward to that of the destructured samples. At $p' = 1500$ kPa, these two lines coincided suggesting that the behaviour of bonded soils has almost changed to that of the destructured material.

CONCLUSIONS

Based on a series of undrained triaxial tests carried on artificially bonded and destructured soils, the following

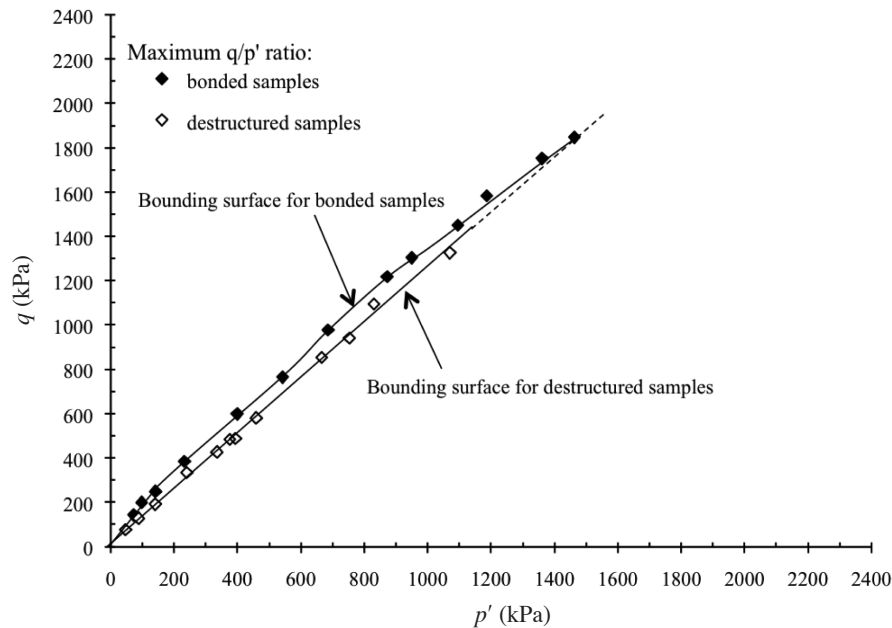


FIGURE 8. Bounding surface for the bonded and destructured samples

conclusions can be drawn from this study. It is clearly seen that the values of maximum deviatoric stress, q_{max} increases with an increase of applied confining stress. This behaviour suggested that there is an increase in area of particles contacts as a result of increasing the surrounding pressure. Therefore, a higher stress is required to overcome the resultant shear stress due to a higher applied confining stress. In terms of shearing at lower effective stress, p' , samples experienced compression at early strain then followed by dilation during shearing behaving like an overconsolidated soils. However, at higher effective stress, p' , soils only experienced compression up to the end of shearing. The bounding surface for destructured soils is represented by straight line which was lower than that of bonded soils. This line is equivalent to stress ratio, q/p' of 1.24 and in terms of q/p' ratios, the bonded samples can sustain a higher limiting stress ratio than the destructured samples at low and intermediate stress levels. However, by increasing the stress level, p' , the maximum values of q/p' ratios of bonded soils decrease and their behaviour become close to destructured soils. This suggests that the bond structures were gradually destroyed due to increase in applied effective stress, p' . Therefore the behaviour of the bonded soils has gradually changed to destructured soils as the structures breakdown.

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